

The Real World of Embankment Settlement

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SUMMARY Four case studies of embankment settlement are examined to illustrate the problems faced most commonly by the practising profession of geotechnical engineers, as distinct from those carrying out research studies in this area. The case studies chosen cover a variety of soil stratifications including thick clay layers and interbedded compressible strata where it is difficult to confidently determine appropriate layering for predictive models. It is shown that conventional analysis methods and standard laboratory testing techniques are often unsatisfactory for interpretation of embankment behaviour, especially for thick layers.

1 INTRODUCTION

In approaching the problem of embankment settlement on compressible soils, the practising geotechnical engineer must compromise between achieving the best technical result and meeting realistic works programme and budget constraints. It is unfortunate that major works programmes of highways departments are most likely to suffer from any technical inadequacies in settlement prediction because of the interdependence of a large number of critical activities. On smaller projects, delays to allow a longer period of consolidation can often be accommodated more easily by re-allocation of funds to other works.

There is strong financial incentive to construct road embankments wherever possible, in preference to bridge structures, which in turn demands that the problem of settlement prediction receives more than cursory attention. By way of comparison, for a two lane rural highway, bridging of soft ground costs around ten times as much as constructing a five metre compacted earth fill and pavement.

Where embankments on compressible soils abut bridge structures, the familiar bump at the junction between the two reveals one of several problems that can develop due to settlement. If the soil movements are large enough, bridge abutment bearings and expansion joints can be damaged and piles over-stressed through excessive bending (due to associated lateral soil movement) and axial forces (due to negative skin friction). Despite several decades of development in the field of soil engineering, the problem of embankment settlement has proved extremely complex. The following brief review and case studies highlight some common difficulties.

2 THE STATE OF THE ART

Before turning to the case histories, it is worth briefly reviewing some current thinking on consolidation theory. In the 20th Terzaghi Lecture, Mitchell (1986) describes several instances where field consolidation behaviour did not follow expected patterns. One of the main problems was interpretation of pore pressure responses beneath embankments on soft clay. Excess

pore water pressures were often found to be very much greater than expected on the basis of settlement observations. One explanation for this phenomenon is that the soil close to drainage boundaries consolidates relatively quickly, producing a skin of very low permeability soil, which retards dissipation of excess pore water pressure from deeper within the consolidating stratum. It would therefore seem that analyses not accounting for layers of significantly different permeability could be seriously in error.

Mitchell also draws attention to apparent soil structural collapse occurring when some soft clays are loaded beyond the effective preconsolidation pressure. This characteristic may be seen clearly in laboratory consolidation curves and leads to two main difficulties in extrapolation to field situations. Firstly, there is a large change in soil compressibility which should be taken into account in developing the settlement model. Simple 1-D consolidation theory with assumed constant compressibility is inapplicable. Secondly, in this critical effective stress range, some soils exhibit time-settlement curves which cannot be interpreted using standard techniques. Such behaviour has significance affecting both time rate predictions and actual magnitude of settlement, (McDonald and Cimino (1984) and McDonald (1985)).

There are clearly problems with interpretation of the consolidation behaviour of some soils, which cannot be satisfactorily solved by approximations to allow simple theory to be used. Nevertheless, in the author's experience, most practical embankment settlement predictions revolve around simple 1-D consolidation theory, with perhaps some averaging of parameters, in recognition of the variability of natural soils (and indeed the variability resulting from standard sampling and testing procedures.

Hawley and Borin (1973) systematically analysed the consolidation process and concluded that the Terzaghi (see Taylor, 1948) consolidation theory, even with modification for secondary compression, was in many cases incapable of providing a reasonable settlement model. They refuted the most fundamental assumptions of the traditional theory and claimed that there was sufficient evidence to support a settlement model in which the

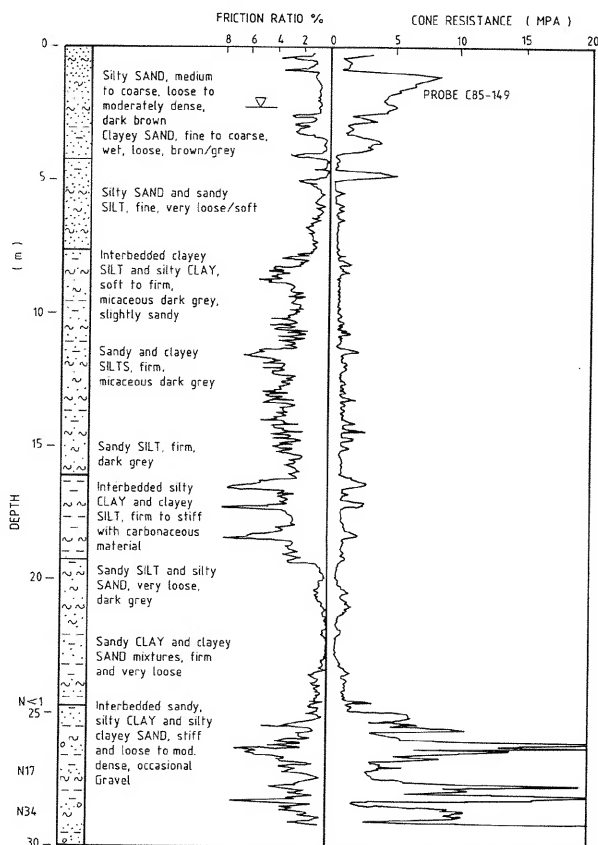


Figure 1 Soil stratification - Site 1

compression-effective stress path followed by a clay was a function of both loading rate and strain rate. It is normally assumed that a unique compression-effective stress path exists, with perhaps some empirical modification for delayed compression or creep, as advanced by Bjerrum (1967). The influence of loading rate (ie. load increment ratio $l.i.r. = \Delta p/p$) on the consolidation behaviour of laboratory specimens is well documented (eg. Leonards and Girault, 1961) but there is less known about the influence of strain rate. Field strain rates are orders of magnitude lower than laboratory rates. The influence of $l.i.r.$ on consolidation is especially important in the case of thick layers where the $l.i.r.$ can vary from large values near the ground surface, to less than one at depth. An interesting conclusion of these authors is that the local magnitude of excess pore pressures does not necessarily indicate whether consolidation is proceeding in a primary manner, a secondary manner or not at all.

The implications of Hawley and Borin's work are far-reaching indeed because most of the settlement models which allow variation of soil compressibility, permeability and coefficient of consolidation to be accounted for (eg. Olson and Ladd, 1979), embody many of the classical consolidation theory assumptions in some way or other. These observations apply equally to radial consolidation theory (Hansbo, 1981) used in the design of sand and wick drain systems to accelerate settlement.

The author is certainly not in favour of abandoning existing settlement methods but the preceding review should serve as a warning for those expecting accurate settlement predictions, even if

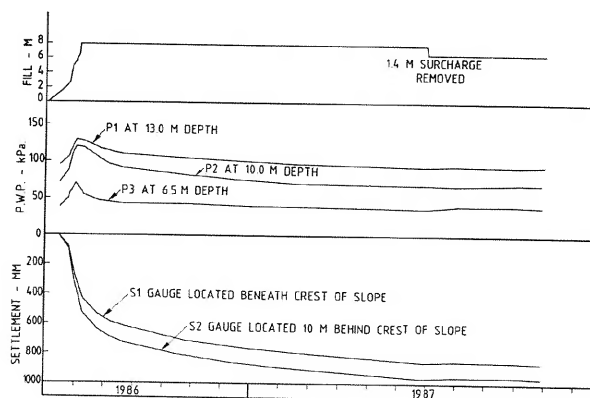


Figure 2 Field observations - Site 1

the methods allow for variations in soil properties spatially or with change in effective stress level. It is worth noting that, whilst Hawley and Borin's Unified Theory appears to account for many laboratory consolidation test departures from traditional theory, its field application appears to be unproven as yet. Furthermore, it cannot take unloading into account, which would seem to be a serious disadvantage for cases involving temporary surcharging.

3 CASE STUDIES

3.1 Site 1

At this site, a 5.5 metre embankment was proposed between two bridge structures across a Recent alluvium flood plain. Because of the expected long term settlements, surcharging to a total height of eight metres was carried out. The top width of the fill was about 14 metres and the side slopes at 2(H) to 1(V).

3.1.1 Soil description

The alluvium comprises loose sands and clayey silts with minor silty and sandy clay layers, the total depth of compressible materials being around 25 metres. The cone penetrometer (CPT) plot in Figure 1 shows the strength of the soil and the irregularity of the friction ratio trace within the clayey silts between depths of 8 to 20 metres illustrates the degree of interbedding. The clayey silts have average properties as shown in Table 1.

Consolidation tests were carried out on 75 mm diameter, 19 mm thick specimens of the clayey silt, with a $l.i.r.$ of 1 but because of the variability of the samples in the thin-walled tubes, it was difficult to decide what constituted a "representative" sample. For the pressure range 100 to 200 kPa, coefficients of consolidation (C_v) ranged between 10 and 60 m^2/yr , based on Taylor's square root of time curve-fitting procedure. The coefficient of volume compressibility (m_v) varied between 0.28 and 0.38 m^2/MPa over the same pressure range. At an effective stress of 200 kPa, the coefficient of secondary compression C_{α} varied between 0.004 and 0.009, where:

$$C_{\alpha} = (\delta h/h) / \log \text{ cycle of time} \quad (1)$$

3.1.2 Settlement prediction

At the time of the geotechnical investigations, the highway alignment and gradeline above natural surface had not been finalised. However, it is

TABLE 1

Soil Properties	Site			
	1	2	3	4
Liquid Limit %	38	74	81	31
Plasticity Index %	7	47	53	12
Per Cent < 75 μ m	64	78	97	51
Per Cent < 2 μ m	18	24	45	16
Water Content %	58	75	71	38
Bulk Density t m ⁻³	1.42	1.54	1.58	1.79

still useful to consider the settlement predictions that were made for an assumed fill height of 4.5 metres.

The approach used was to perform the settlement calculations in three different ways and to average the values obtained. This might not seem a sophisticated technique but, in the author's experience, is a common one used in the solution of various geotechnical problems. The first solution involved a simple oedometer calculation using the formula:

$$S_{\text{Oed}} = \sum (m_v \cdot \Delta p \cdot \delta h) \quad (2)$$

where: Δp = vertical stress increase in the layer of thickness δh

The second solution used the well known consolidation equation:

$$S = \frac{C_c \times \delta h}{(1 + e_o)} \times \log_{10} \left[\frac{(p_o' + \Delta p)}{(p_o')} \right] \quad (3)$$

where: C_c = compression index
 e_o = initial void ratio
 p_o' = in-situ vertical stress

A separate calculation of compression in the region below the effective pre-consolidation pressure (p_o') was not performed initially because the final vertical stress was well above this value and also because consolidation curves showed considerable variability.

The third technique used was to divide the sub-soil into layers according to the penetrometer cone resistances and to apply the method of Belshaw (1973). This method is strictly supposed to apply to clays but was still used here because of the fine nature of the soil, the high soil water content and the laboratory evidence of high compressibility. The method has appeal in that it utilises continuous traces obtained over the whole depth of the deposit and is simple to apply. In highly stratified deposits, there is always some doubt as to whether discrete laboratory samples give a satisfactory representation of the entire layer. A potential problem with the Belshaw method however, is that if very loose sandy layers with similar cone resistance to clay layers are inadvertently included in the summation, settlements can be over-estimated.

Calculation results were as follows:

$$\begin{aligned} S_{\text{Oed}} &= 860 \text{ mm} \\ S_{\text{(eq.3)}} &= 800 \text{ mm} \\ S_{\text{(Belshaw)}} &= 680 \text{ mm} \end{aligned} \quad \text{Average: 780 mm}$$

Similar calculations were later performed for a 6.5 metre high fill, with a predicted settlement of 1080 mm (the final fill height was in fact close to 8 metres).

Creep settlements, based on maximum laboratory C_v values indicated that around 180 mm of further settlement would occur over 10 years.

Laboratory C_v values yielded 90 per cent primary consolidation times ranging between 6 months and 3 years.

3.1.3 Field measurements

Figure 2 summarises the actual settlements and total pore water pressures (pwp) recorded at depth below the embankment. It can be seen that, about 8 months after filling, piezometer readings had returned to their static values although the rate of settlement continued at around 200 mm per year. Although not shown in Figure 1, two inclinometer casings were installed to depths of 27 metres at the toes of the fill to check on stability during construction and to assist with interpretation of immediate settlements as a result of lateral soil movements. Maximum lateral soil movements of 72 mm were recorded at the front toe of the fill at a depth of 5 metres. At the side of the fill, lateral movements were about 40 mm, at 7 to 8 metres depth.

3.1.4 Post mortem

If settlement due to lateral soil movements is ignored, which seems reasonable in this case, measured consolidation settlements of about 900 mm occurred. Extrapolating the above predicted settlements somewhat to obtain the settlement for an 8 metre fill, a figure of about 1300 mm is obtained. Failure to account for over-consolidation of the soil probably accounts for most of the over-estimate. However, settlements continuing after excess pore pressure dissipation were substantially greater than predicted. It is interesting that the Belshaw computation makes no separate allowance for over-consolidation but uses penetrometer cone resistances directly. Presumably, any over-consolidation effects on settlement are reflected in those cone resistance values. Of the three prediction methods used, the Belshaw method gave results nearest the measured settlements.

At this point the surcharge has been removed to study whether significant additional settlements will develop. Because of the settlement that had occurred, only 1.4 metres of the original 2.4 metres of surcharge placed above the finished embankment level was removed. There are indications that, after an initial cessation of settlement following surcharge removal, settlement is continuing at a reduced rate, although it is too early to be sure of likely long term behaviour.

3.2 Site 2

The primary aim at Site 2 was to raise an embankment above flood level through a swamp. Control of long term settlements was not a serious issue because of the absence of waterway structures near the area of the most compressible soils. A construction platform was provided by means of felled timber placed to form a mat over the extremely soft surface soil. Sand was then spread over the mat to give a reasonably level surface. The thickness of this corduroy road was about 1

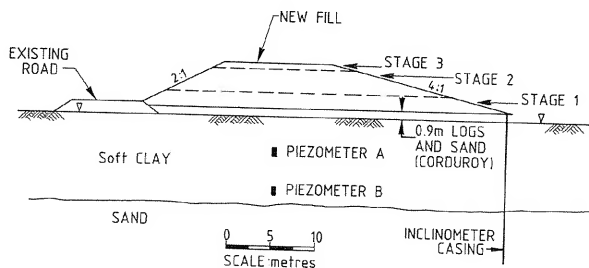


Figure 3 Fill cross-section - Site 2

metre. Figure 3 shows a cross-section of the new fill.

3.2.1 Soil description

Figure 4 shows the soil profile. The bore and CPT were taken from the level of the road. Nearly all settlement was expected to occur in the soft silty clay up to 9 metres deep at the worst location. Average clay properties are given in Table 1.

Laboratory consolidation test data for the relevant stress range were:

$$\begin{aligned} m_v (55 - 110 \text{ kPa}) &= 1.85 \text{ m}^2/\text{MN} \\ C_v (55 - 110 \text{ kPa}) &= 0.73 \text{ m}^2/\text{year} \end{aligned}$$

Creep properties were not obtained from the consolidation tests.

3.2.2 Settlement prediction

In this case, settlement prediction was elementary, mainly based on a simple 1-D oedometer calculation. A check on this result was made using eq.3 above, with a value of the compression index (C_c) determined through a water content correlation given by Nishida, 1956 (eq.4).

$$C_c = 0.0054 * (2.6 * w - 35) \quad (4)$$

where: w = water content in per cent

Calculations for a 5.2 m fill gave the following settlement estimates.

$$S_{\text{oed}} = 1665 \text{ mm}$$

$$S (\text{eq.3}) = 2220 \text{ mm}$$

No account of possible over-consolidation was taken in applying eq.3. However, the CPT plot indicates significant over-consolidation towards the surface, in which case the second estimate above is likely to be too large. Also, both of the above figures should be over-estimates because of the reduction in applied stress due to submergence of the lower part of the fill. This affects the estimates differently because of the way in which the two settlement equations are formulated. If submergence is allowed for, S_{oed} above is reduced by around 20%, whilst $S (\text{eq.3})$ is reduced by only 11%.

The author subsequently computed settlements according to the Belshaw method using the CPT cone resistances and obtained a value of 450 mm. It should be noted that CPT testing was carried out from the old road because of very soft conditions where the new road was to be built. The over-consolidation indicated by the CPT trace could therefore be due to the influence of the previous filling of around 2 metres.

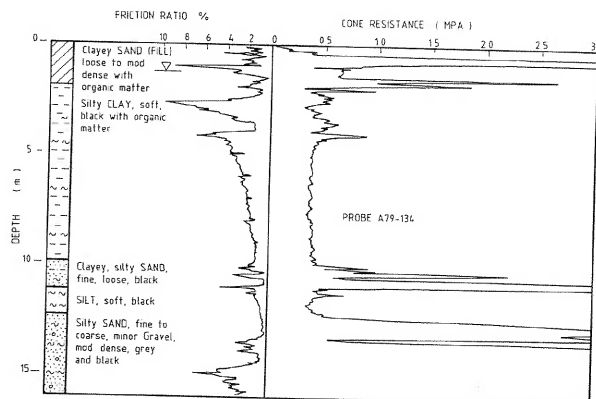


Figure 4 Soil stratification - Site 2

3.2.3 Field measurements

Figure 5 shows the results of settlement and pore water pressure (pwp) measurements beneath the fill. At this time, settlements of 2150 mm have occurred.

During the stage 2 filling, the toe inclinometer casing underwent a disproportionate lateral movement 4.75 m below the original ground surface. Filling was suspended temporarily and lateral movements abated. The strength of the corduroy mat below the fill probably prevented collapse of the embankment at this time. A maximum lateral movement of 580 mm has been recorded to the present time, at a depth of 4.3 m.

Excess pwp has completely dissipated near the bottom of the clay but piezometer A still registers an excess of around 73 kPa over the original static value. If allowance is made for settlement of the piezometer itself, there is still an excess pwp of about 60 kPa. This indicates about 20% dissipation, allowing for the gradual reduction in effective vertical stress due to partial submergence of the fill.

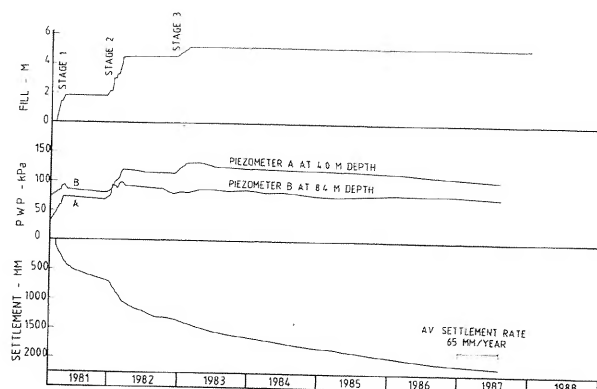


Figure 5 Field observations - Site 2

3.2.4 Post mortem

The indications are that the soil beneath the existing road fill is over-consolidated to some degree. Hence, settlements computed on the basis of this information are likely to be too low. This is a case where attempts to carry out a reliable settlement prediction in advance are practically impossible.

Applying standard 1-D consolidation theory to the

problem indicates that, for around 20% excess pwp dissipation at the mid-layer depth, consolidation should be only be 40% complete. In other words, more than 5.4 metres of settlement could be expected to occur, compressing the entire 9 metre compressible layer to 40% of its original thickness! The settlement curve characteristic shows this to be highly unlikely. Discrepancies of this order highlight the inadequacies of the simple 1-D theory for some practical cases.

3.3 Site 3

A large culvert crossing beneath a major urban freeway embankment at this site dictated that long term settlements should be minimised as much as possible. Figure 6 shows a plan of the site. Initial proposals were that consolidation would occur fairly quickly but settlement and pore water pressure monitoring showed that such expectations were very much in error.

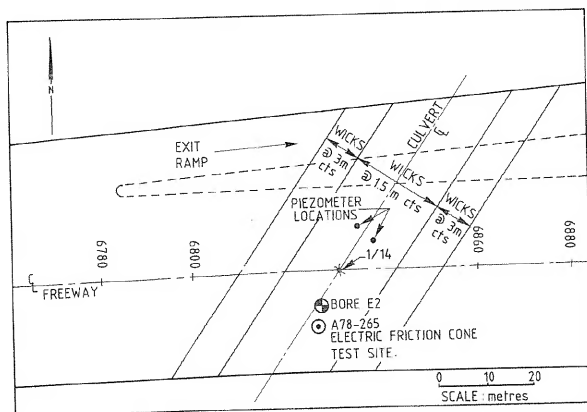


Figure 6 Plan of embankment - Site 3

Wick drains were installed at the culvert location to accelerate settlements. The rate of settlement increased dramatically but pore pressures did not drop as quickly as planned and finally, because of works programme constraints, it was decided to proceed with construction of the culvert and the freeway itself.

3.3.1 Soil description

The soil at the site is an alluvial silty clay known locally as Coope Island Silt. Figure 7 shows a CPT trace and bore log of the strata. This material exhibits perplexing characteristics both in the field and the laboratory, when loaded above the effective pre-consolidation pressure (McDonald and Cimino, 1984). Average soil properties are shown in Table 1.

During laboratory consolidation tests, large variations in m_v and C_v values were noted and there were problems with interpretation of many test curves, especially in relation to deciding when further load increments could be added. In most cases, for load increments straddling the p_c value, a sharp increase in compressibility (m_v^c) and a corresponding fall in coefficient of consolidation (C_v) was observed. Peak m_v values of $1.5 \text{ m}^2/\text{MN}$ were calculated along with C_v values of about $0.5 \text{ m}^2/\text{year}$, but with large variability.

Preliminary laboratory tests gave C_α values of around 0.009. Later tests gave values of from 0.012 to 0.021, with an average of 0.016 at the

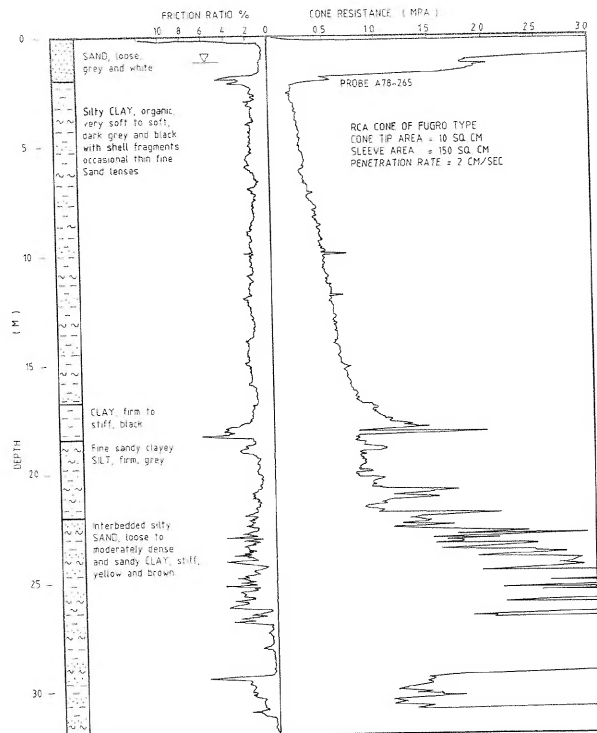


Figure 7 Soil stratification - Site 3

relevant pressure. In several tests the time-settlement curves did not allow definition of primary and creep phases.

3.3.2 Settlement prediction

With hindsight, it would seem that some considerable fortitude was required to make settlement predictions for this site. At this and similar adjacent fill sites, predictions were subject to substantial revision as field monitoring results became available. The original predictions are given here for a fill height of 4 metres:

S_i	=	135 mm	(immediate settlement)
S_c	=	1040 mm	(consolidation settlement)
S_{cr}	=	20 mm/year	(creep settlement)

Settlement predictions were based mainly on a consolidation equation of the form of eq.3, except that allowance was made for over-consolidation deduced from laboratory consolidation tests. Primary consolidation in these tests was assessed using Taylor's square root of time graphical method. Often, for pressure increments straddling the p_c value, compression vs log time plots could not be successfully interpreted.

Use of vertical drains to accelerate consolidation should not, in theory, change the predicted settlements. The wicks were expected to reduce the time for primary consolidation to about 6 months.

3.3.3 Field measurements

Figure 8 shows settlements and pore water pressures beneath the centre of the fill. Some piezometer lines were damaged during construction works. Settlements were monitored by survey markers on top of the fill initially and later by survey of points marked on kerbs.

Total settlements were not measured directly at

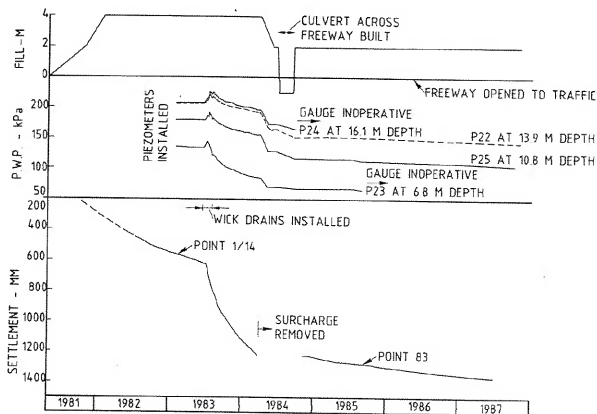


Figure 8 Field observations - Site 3

this site but inferred from virtually identical adjacent areas where hydraulic settlement gauges had been installed prior to filling; hence the dashed line for early settlements on Fig.8. Point 83, on a median kerb, is very close in plan to where the earlier point 1/14 had been located on the fill.

If settlements are replotted to show rate of settlement against time, it can be shown that, although settlement rates declined steadily from a peak following wick installation, for a period of about three months before surcharge removal, the rate of settlement remained essentially constant at almost 500 mm/year.

The average rate of settlement of point 83 since freeway construction is 54 mm/year. A similar point, 50 metres away, and remote from the influence of the wicks, has experienced exactly one third the rate of settlement of point 83.

3.3.4 Post mortem

Almost two years after construction, excess pore water pressures had shown little dissipation throughout the most compressible layers, despite a settlement of 600 mm, 60 per cent of that estimated for primary consolidation.

Wick installation accelerated settlement rates initially but, after some time of operation, rates did not continue to diminish. By this time of course the predicted settlements had been exceeded and the shape of the curve showed that considerable further settlements could be expected. Some of the increased settlement could have been brought about by destructuring of the clay during the wick placement process as the mandrel of the equipment penetrated the soil at 1.5 metre centres. A general rise in pore water pressure throughout the layer at the time of placing the wicks indicated an increase in total stress levels due to the process.

Another factor to consider is that of strain rate, which, according to Hawley and Borin, has a significant affect on the compression-effective stress relationship. In this case, three different strain rates would need to have been considered in extrapolating from the results of laboratory tests to the field case, with and without the wick drains installed.

Clearly, this is a case where application of simple consolidation theory was not appropriate. However, the current literature does not indicate that

modifications for variable permeability and compressibility, large strains and the like, would have provided a satisfactory theoretical settlement model for this complex problem.

Whatever the mechanism, it would appear that the wicks were not permitted to operate for a sufficient period before surcharge removal, leaving a greater maintenance problem at this location than at adjacent areas without wicks.

3.4 Site 4

There was no reason to expect problems at Site 4, which comprised mostly sands, except for the existence of a layer of relatively fine-grained soil about 5 metres thick, sandwiched between layers of loose to moderately dense sands extending to depths of 25 metres.

Since relatively minor settlement of embankments abutting bridge structures can cause problems such as development of large negative skin friction forces on piles and significant abutment distortion, attention was focussed on the characteristics of the fine-grained soil layer. Finally, surcharging was adopted to ensure that future settlements would not produce problems.

The embankment proposed was to be 7 metres high with a top width of 12 metres and side slopes of 1.5 (H) to 1 (V). A 3 metre high surcharge placed above the design level had side slopes of 1 to 1.

3.4.1. Soil description

Figure 9 shows bore and CPT information from the site. The fine-grained soil between about 12 m and 17 m varied in character from a micaceous low plasticity silty clay to a clayey silt, with inter-bedded loose sand lenses. The fine-grained soil has average properties as shown in Table 1.

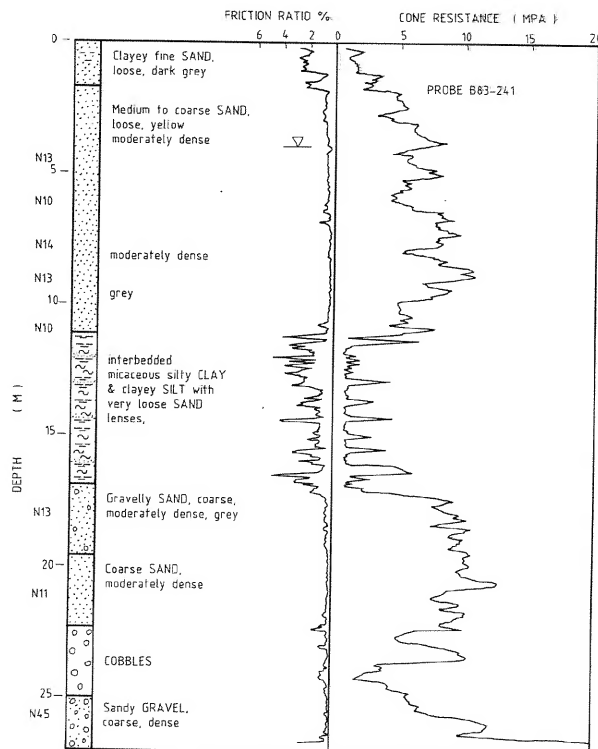


Figure 9 Soil stratification - Site 4

Over the stress range from the in-situ value to that following embankment loading, the consolidation parameters m_v and C_v were almost constant, with:

$$\begin{aligned} m_v &= .23 \text{ m}^2/\text{MN} \\ C_v &= 70 \text{ m}^2/\text{year} \end{aligned}$$

The coefficient of secondary compression C_{α} was 0.005

3.4.2 Settlement prediction

In contrast with the previous case studies, immediate (elastic) settlement was expected to form a significant proportion of the total because of the relatively small consolidation and creep settlements. Immediate settlements are not usually a problem with embankment behaviour and therefore, calculations are often approximate. In this case, the fill was approximated as a rectangular footing on a finite layer. A uniform elastic modulus of 20 MPa was assumed, based on CPT cone resistances of around 6 MPa. Elastic settlements of 116 mm were calculated for the centre of the fill and 105 mm for the bridge abutment position.

Consolidation settlements were estimated using the Belshaw method and on the basis of eq.(2) using the laboratory m_v value, assuming that the compressible layer between about the 12 m and 17 m depths comprised 60 percent silty clay, for which the above m_v value was applicable. Results of these calculations were as follows:

$$\begin{aligned} S_{\text{Oed}} &= 84 \text{ mm} \\ S \text{ (Belshaw)} &= 110 \text{ mm} \end{aligned}$$

Consolidation settlements were expected to occur extremely quickly (< 1 week) because of the high C_v value and the existence of closely spaced sand lenses separating the silty clay layers.

3.4.3 Field measurements

Figure 10 shows the settlements recorded by hydraulic gauges located 3 metres behind the abutment centreline beneath the crest of the surcharged fill. Five months after the end of construction, settlements had all but completed, except for a sudden increase in settlement of about 75 mm in early 1987, which coincided with pile driving at the abutment.

3.4.4 Post mortem

It would appear that the predicted settlements matched the observed values closely. However, the field measurements do not differentiate between immediate and consolidation settlements. For example, it is possible that immediate settlements were over-estimated and consolidation settlements under-estimated. In the author's experience with many embankment settlement studies, this possibility is most likely. Sufficient time has not elapsed to verify creep settlement predictions.

In this instance, laboratory and field behaviour were able to be interpreted without much difficulty and the use of surcharge appears to have been successful. Only minor future settlements are expected.

An unexpected development was the increase in settlement after surcharge removal, presumably due to vibratory compaction as a result of pile driving. At an adjacent abutment fill, similar

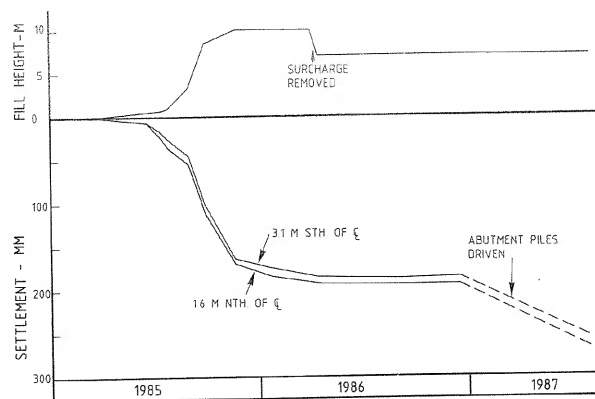


Figure 10 Field observations - Site 4

effects were observed at a different time but again coinciding with the time of pile driving.

4. CONCLUSIONS

In deep interbedded deposits of alluvium it is possible to obtain a wide variation in calculated settlements. For such soils, the practitioner is faced with the problem of making several somewhat arbitrary decisions in assigning soil parameters. Normal laboratory samples are too small to correctly represent mass consolidation characteristics. It is possible of course to conduct a more rigorous sampling and testing programme and employ more complicated theory, but there is in reality seldom time or money available for such an approach. A practical solution, if there is sufficient time available, is to conduct a trial embankment programme and use fairly simple

theory to back-analyse the results. However, it must be realised that this approach leads to site-specific results, which may not be satisfactorily translated to other embankment sites.

In many cases of compressible thick inter-bedded alluvial strata, it is difficult to be sure that the soil tested in the laboratory is representative of a particular layer or sequence. Even if this problem is overcome, the actual laboratory test behaviour may not be satisfactorily interpreted using accepted standard procedures.

Of the case studies presented, only one fell into the category of satisfactory settlement prediction. However, in this latter case, the absence of individual layer compression data such as pore pressure and immediate compression information, may have masked some inaccuracies in the prediction.

With the current state of the art in this area of geomechanics, the author cannot foresee in the near future any significant improvements to present practice. What is perhaps most important for those involved with these problems, is to gather field data from as many sites as possible, so as to anticipate the sorts of problems likely to occur at each new site. Such an awareness is an important ingredient in the decision making process for roadworks projects.

5. ACKNOWLEDGEMENTS

The author wishes to thank the RCA field technicians who have made the collection of

embankment performance data possible. This paper is presented with the permission of Mr. I.F.X. Stoney, Chairman and Managing Director of the Road Construction Authority of Victoria. The views expressed are those of the author and do not necessarily reflect those of the Road Construction Authority.

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